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DESIGN OF WIND BRACING

By Edward Smulski *

Synopsis

The object of this paper is to describe proper wind bracing for modern tall structures, and to give formulas for their design. Special attention is given to structural steel buildings and to connections required between the horizontal steel beams and the steel columns forming the wind-resisting frames. The most useful connections are described and illustrated, and formulas are given for their design.

The method of computing bending moments and shears due to wind pressure is not a part of this paper.

Wind-Resisting Frames

In skeleton buildings resistance to wind pressure is usually provided by frames placed, when possible, at right angles to the walls upon which the wind acts. Each frame consists of columns and horizontal beams framed into the columns and rigidly connected with them.

Each building must have at least two sets of wind-resisting frames, placed, if possible, at right angles to each other, in order to resist wind pressure acting from any direction. For instance, a rectangular building facing east or west should be provided with a set of frames extending east and west to resist the pressure acting on the north or south wall, and with a set of frames extending north and south to resist pressures on the east or west wall.

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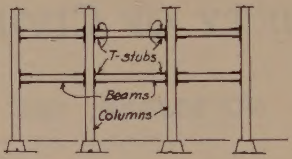
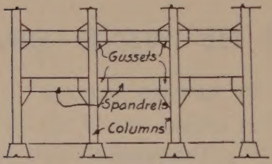
Wind-resisting frames are often placed at each column line, in which case most of the frames are interior frames, consisting of interior columns and floor beams, as shown in Fig. 1*a*. In such frames it is necessary to use connections between beams and columns which can be easily concealed in the floor system. In other designs only a limited number of frames are used, which are then concealed in the walls. Such frames consist of wall columns and spandrel beams, as shown in Fig. 1*b*. Since the spandrels and their connections are concealed in the wall, there is much greater freedom for selecting proper types of connections.



(a) T-stub Connections



(b) Gusset Connections

FIG. 1.—TYPICAL WIND-RESISTING FRAMES

BENDING MOMENT AND SHEARS IN WIND-RESISTING FRAMES

The bending moments in a wind-resisting frame due to wind pressure are of the general character shown in Fig. 2. The maximum bending moments in each member act at the joints, and reduce to zero in the central portion of the member. In addition, each member is subjected to shear and direct stress.

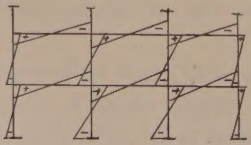


FIG. 2.—BENDING MOMENTS IN FRAMES FOR WIND ONLY
Wind from the left

Since the members of the wind-resisting frame are also members of the skeleton carrying the vertical loads, each member is subjected to bending moments and shears due to live and dead load, as shown in Fig. 2*a*. Before designing the wind bracing it is necessary to compute separately the bending moments and shears due to wind pressure and due to the vertical loads, and to combine them so as to get the most unfavorable combinations. In combining bending moments use dead and live load bending moments when they are of the same size as those for the wind, and dead load only when of opposite size.

In Fig. 2*a* the upper diagram shows the bending moments in the horizontal member for dead and live load. The lower diagram shows by straight lines the bending moments for wind only, and by the curves the combined bending moments for vertical loads and wind, for which wind bracing must be designed. It should be noted that, when the wind pressure acts from the right, the combined bending moments at the left end of the horizontal member are negative and at the right end positive. The reverse is true when the wind acts from the left.

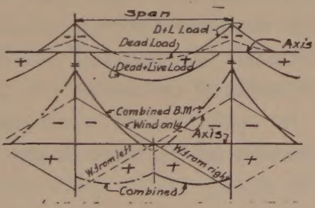


Fig. 2a.—Bending Moments in Horizontal Member for Vertical Loads, Wind and Combined

Wind Bracing for Reinforced Concrete Structures

The method of providing wind bracing for reinforced concrete structures is very simple. After the combined bending moments, shears and direct pressures for the vertical loads and the wind pressure are computed, the problem resolves itself into providing members of sufficient dimensions and with sufficient amount of reinforcement to resist the expected bending moments, shears and direct stresses.

The arrangement of reinforcement in a beam used as a brace differs in several respects from that of a beam carrying only vertical loads. (*a*) At the column the wind brace must be able to resist both positive and negative bending moments, depending upon the direction of the wind. This requires positive reinforcement at the columns, which must either be properly anchored in the column or extended into the adjoining span. (*b*) In a wind brace the zone of negative bending moments extends much farther from the column than in an ordinary restrained beam. (*c*) Since the wind pressure produces in each span constant shear, it may be necessary to provide diagonal tension reinforcement throughout the whole length of the brace.

When required, the spandrel beam used for wind bracing may be extended above the floor level to provide additional depth. Since the bending moments are largest at the columns, haunches may be found useful to reduce the compression stresses. Where the available depth and width are not sufficient, compression reinforcement may be used.

Columns forming a wind-resisting frame are subjected to direct stress and bending. Since the allowable unit stresses for a combination of vertical loads and wind pressure are larger than for vertical loads alone, the size of the columns as required for vertical loads may be sufficient, also, to resist wind stresses. Occasionally additional reinforcement may be required to resist bending moments.

For the design of the members of wind-resisting frames the formulas used in ordinary reinforced concrete design should be used.

Structural Steel Buildings

For structural steel buildings with wind-resisting frames without diagonals considerable ingenuity may be required to design effective and at the same time simple wind bracings.

The design of the columns is not affected to any great extent by the wind pressure. Since the allowable stresses for combined vertical loads and wind pressure are larger than for vertical loads alone, a large part of the increase in pressure is usually offset by the increase in allowable stresses. The effect of wind pressure upon the columns is not large except in very tall and narrow buildings.

The horizontal members of wind-resisting frames are affected very much more by wind pressure. The bending moments and shears in the horizontal members for a combination of wind pressure and vertical loads are appreciably different from those for vertical loads alone.

To be effective a wind-resisting frame must be provided with rigid connections between the vertical and horizontal members. With rigid connection the vertical loads produce negative bending moments at the columns. These are increased by negative bending moments, due to wind pressure.

The most difficult problem in structural steel buildings is to provide connections between the horizontal members and vertical members of a wind-resisting frame sufficiently strong and rigid to resist the maximum bending moments acting at the edge of the columns.

The type of connections to be used between the horizontal and vertical member of a wind-resisting frame depends upon the design of the members and their position in the building. In this paper the connections are divided into three groups: (1) T-section connections; (2) spread gusset connections for built-up girders; and (3) gusset connections for rolled sections. These three types are discussed under separate headings.

1. T-Section Connections

The T-section connection, shown in Fig. 3, consists, at each end of the beam, of two T-sections placed at the top and bottom of the beam, and of connecting angle (or angles) riveted to the web of the beam and to the column. The T-sections and their connections are designed to resist the bending moments, while the connecting angles take care of the vertical shears.

The T-sections are formed from I-beams, or girder beams, by splitting through their webs. The flange of the T-section is riveted to the column, while the web is riveted to the flange of the beam. The depth of the beam used for forming the T-section depends upon the number of rivets required for the connection of the web of the stub to the beam. The weight of the beam depends upon the thickness of the flange of the stub required to prevent its distortion when subjected to the bending moments produced by the tension in the rivets.

When two lines of rivets in the flange of the T-stub are sufficient to supply the required tension, the T-section is formed of standard I-beams. In such case the number of rivets is limited to eight, *i.e.*, four rivets in each line. When T-stubs are used as connections for heavier beams, eight rivets are not sufficient. It is then necessary to use four lines of rivets in the flange of the T-stub, which is possible when the stub is formed of wide flanged girder beams. The heaviest available section is usually necessary to supply a thickness of flange sufficient to resist the bending moment produced by the four lines of rivets.

Ordinarily the stubs and the connection angle (or angles) are shop riveted to the column, and field riveted to the beam. To facilitate erection, the outstanding webs of the stubs are spread by bending them outward to facilitate the insertion of the beam between them. In some designs, in order to avoid the necessity of spreading the webs, the clear distance between the webs of the stubs is made larger than the depth of the beam. The extra space is then filled by a filler inserted after the beam is in place.

The connection is subjected to positive or negative bending moments, depending upon the direction of the wind pressure. The bending moments produce shear in the rivets connecting the web of the stubs to the flange of the beam. The total shear in all rivets on the top of the beam is equal to the total shear in the rivets at the bottom of the beam, but it is of opposite sign. Since the resisting moment of the rivets, equal to the total shear in one set of the rivets multiplied by the depth of the beam, must be equal to the bending moment, the total shear in rivets equals bending moment divided by depth of beam, $\frac{M}{d}$, (see Fig. 3.)

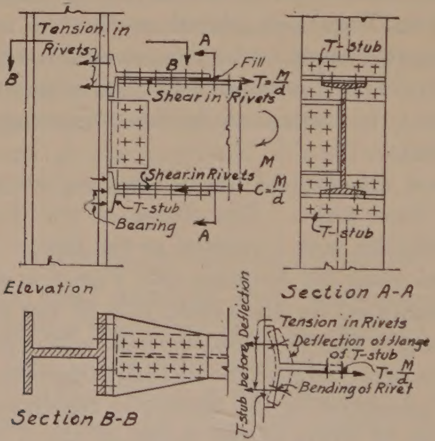


FIG. 3.—T-STUB CONNECTIONS

The tensile stresses transferred from the web of the stub to the column are resisted by tension in the rivets connecting the stub to the column. The compression stresses are resisted by the bearing of the stub on the column.

Since the stresses in the web are not on line with the resisting stresses in the rivets, bending moments are produced in the flange of the stub. The thickness of the flange must be large enough to resist these bending moments without exceeding safe working stresses. The tensile stresses in the rivets are eccentric with a degree of eccentricity depending upon the rigidity of the flange of the stub. The more rigid the flange, the smaller is the eccentricity of the tension in the rivets.

In Fig. 3 the flange of the T-stub, with the forces acting upon it, is shown to a larger scale. The exaggerated deflection of the flange is shown by dash lines. This also shows the bending in rivets caused by the deflection. The smaller the deflection of angles, the smaller will be the bending in the rivets.

The connection is designed as follows:

1. Compute the bending moments in the horizontal member of the frame for vertical loads and for the wind pressures acting from the right and from the left. Combine the bending moments so as to get the largest possible positive and negative bending moments at the junction between the beam and the column. The maximum negative bending moment is obtained for wind acting from one direction, while the maximum positive bending moment acts for wind from opposite direction. The maximum negative bending moment is always larger than the maximum positive bending moment, because it is composed of the negative bending moment for wind and the bending moment for vertical loads, which is always negative at the columns.

2. Decide upon the method of erection to determine which rivets are shop driven and which are field driven.

3. The number of rivets required to connect each web of the stubs to the flanges of the beam equals the maximum bending moment (M) divided by the depth of the beam (d) and by the unit stress of one rivet in single shear. The number of rivets should be increased when they pass through a loose filler.

4. The number of rivets connecting the upper T-stub to the column is governed by the maximum negative bending moment which produces tension in these rivets and bearing of the lower stub on the flange. The tension to be resisted by the rivets equals the negative bending moment divided by the depth of the beam. The number of rivets is obtained by dividing the total tension by the tension value of one rivet. If the number of rivets is limited (as is usually the case), the total tension divided by the available number of rivets gives the stress to be resisted by one rivet. This determines the size of one rivet.

5. Find the required thickness of the flange of the T-stub. It may be considered as a beam supported by the rivets and loaded by the pull exerted on the web. The effective span may be taken as the distance between the centers of the lines of the tension rivets. The maximum bending moment acts at the junction of the web and the flange of the stub. The required thickness of the flange may be obtained from the formula —

$$t=\sqrt{\frac{6M}{lf}}$$

in which t = the thickness of flange in inches
M = the maximum bending moment in inch-pounds
l = the length of the stub in inches
and f = the unit stress in pounds per square inch.

6. Investigate the tensile and compressive stresses in the webs of the stub.

7. Design the connection of the lower T-stub to the column. This is governed by the positive bending moment at the column, which is always smaller than the negative bending moment. The lower stub, therefore, needs fewer or smaller rivets than the upper stub, and the thickness of its flange also may be smaller. Of course, to facilitate fabrication, both stubs may be made the same.

8. Design the required shear connection. Usually, to facilitate erection, a single angle 8″×8″×½″ is shop riveted to the column by two rows of rivets, and then field riveted by two rows of rivets to the web of the beam. When it is not possible in the column to get two lines of rivets on one side of the center line of the beam, two angles are used, placed so that the backs of their outstanding legs touch. One line of rivets each connects the angles to the column, while the beam is riveted to one side of the two outstanding legs by two rows of rivets.

Special Method of Resisting Negative Bending Moments for Vertical Loads. — Negative bending moments due to vertical loads acting on one side of the column are partially or wholly balanced by the negative bending moments on the other side of the column. Therefore, when a beam is subjected to negative bending moments for vertical loads, in addition to the negative bending moment due to wind pressure, a part or the whole of the negative bending moment for vertical loads may be resisted by transferring it directly from the beam on one side to the beam on the other side without transferring it first to the column.

For a construction, using reinforced concrete floor construction, and in which the steel beam is encased in concrete, the negative bending moment may be resisted in part by top reinforcing bars placed on each side of the column, and by the compression resistance of the concrete. In this way the number of tension rivets required to connect the upper T-stub to the column may be reduced. If the concrete surrounding the steel beam is not sufficient to resist compression, it may be considered as resisted by the bottom flange of the beam and transmitted by shear to the web of the lower T-stub, and finally by bearing to the column.

The tension in the rivets connecting the upper T-stub to the column may also be reduced by the design shown in Fig. 4, where the tension from one side of the column is transferred to the beam on the other side of the column by two straps, one on each side of the column, riveted to the webs of the two upper T-stubs riveted on both sides of the column. The resistance of the straps in tension equals the shearing value of the rivets connecting the strap to the web.

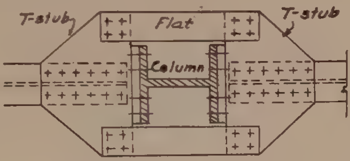


Fig. 4. — Special T-Stub Connections

Angle Connection. (See Fig. 5.) — Where the bending moments due to wind pressure are small, the wind-bracing connection between the beam and the column may consist of two angles or channel stubs instead of T-stubs previously described. The angles and their connections to the flanges of the beam and the column are computed to resist bending moments, while the vertical shear is resisted by an angle (or angles) connecting the web of the beam to the column.

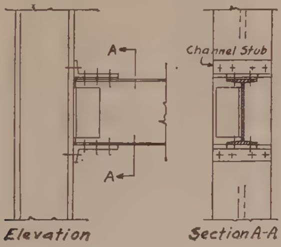


Fig. 5. — Channel Stub Connections

It is of great importance to make the thickness of the angle, or of the leg of the channel stub, large enough to prevent its distortion due to the bending moment caused by the eccentricity of the connection. The vertical leg of the angle or the stub may be considered as a cantilever loaded by the tension in the rivets. The maximum stresses will act at the fillet of the angle. The eccentricity may be taken as the distance of the center of the rivet from the nearest face of the angle. The thickness of the angle is found in the same manner as explained in connection with the T-stub. If the thickness of the angle (or stub) is not large enough, the angle becomes distorted in the manner shown in the failure of the building in the Florida hurricane.

When the required length of the leg to be riveted to the flange of the beam is larger than the leg of the largest available angle, the angles used as bending moment connections may be replaced by stubs obtained from channels of proper depth and proper weight by splitting them along the web.

The number of rivets required to connect the angles or stubs of channels to the flange of the beam is equal to the bending moment at the edge of the column divided by the depth of the beam, and by the value of one rivet in single shear.

The tension to be resisted by rivets connecting the angle to the column is equal to the total shear resisted by the rivets connecting the other leg to the flange of the beam. In some textbooks the tensile stresses in rivets are computed by dividing the bending moment at the column by the distance between the top and bottom line of rivets connecting the angles to the columns. This is obviously incorrect, as can be seen by referring to Fig. 6. The connection at the column must resist the original bending moment M plus the moments M_1, caused by the eccentricity of the connections.

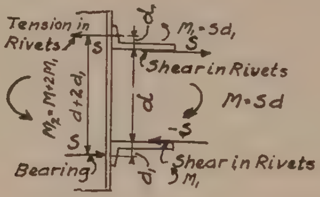


FIG. 6. — STRESSES IN CHANNEL STUB

2. SPREAD GUSSET CONNECTIONS

The Spread Gusset Method is illustrated in Fig. 7. In this method the beam is provided at each end with a bracket formed by a gusset extending above the beam, below the beam or both. These three arrangements are shown in the figure. The bending moments and shears for which the beam should be designed are also shown in the figure. The heavy solid line shows the combined bending moment when wind pressure acts from the right, and the heavy dash and dot line when pressure acts from the left.

It is evident from the figure that the bracket is designed for the negative bending moment acting at the edge of the column. The beam between the splices is designed for negative and positive bending moments.

The spread gusset method is used mostly for spandrel beams, because there the bracket can be concealed in the spandrel wall. The different designs of gusset connections often used in practice are shown in Figs. 8 to 18, inclusive.

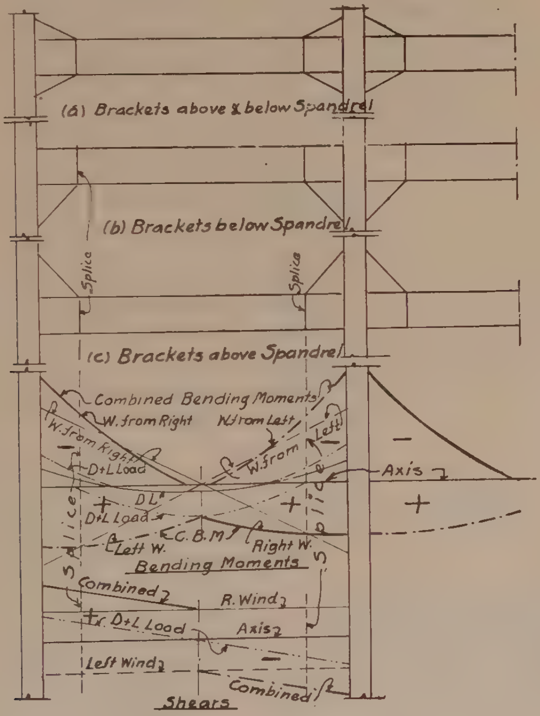


FIG. 7. — GUSSET CONNECTIONS

Gusset Connection with End Angles. — In the designs shown in Figs. 8 to 10 the gussets are connected to the columns by vertical end angles, and to the beam by splice plates and flange angles which overlap each gusset and are riveted to it. These designs may be used when the beam is connected to the face of the column or to its web.

The bending moment acting at the edge of the column is transmitted from the gusset to the end angles by shear in the rivets connecting the gusset to the angles. The intensity of the shear is shown in

Fig. 8. The end angles, in turn, transmit the bending moment to the column by tension in the rivets connecting the angles to the column, and by bearing of angles against the column. The gusset must be thick enough to resist all stresses, particularly at intermediate points. It may be strengthened along the edges by stiffener angles. At the splice of the gusset and the beam the web splices may be designed to

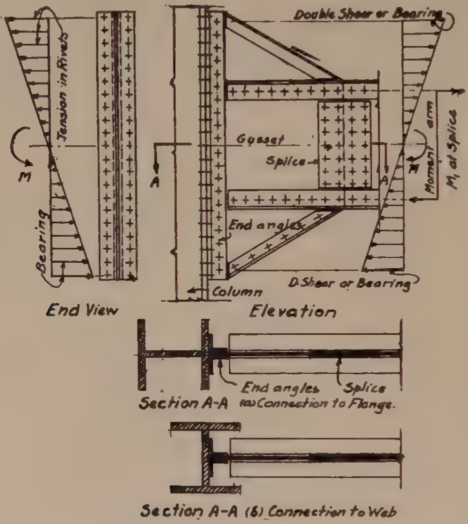


FIG. 8. — GUSSET CONNECTION WITH END ANGLES

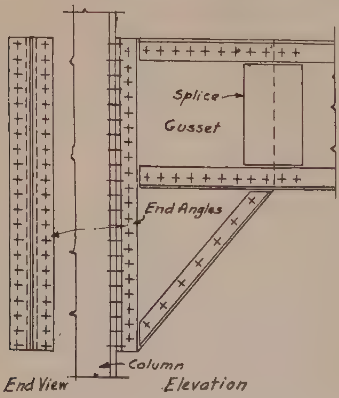


FIG. 9. — GUSSET CONNECTION WITH END ANGLES (GUSSET BELOW)

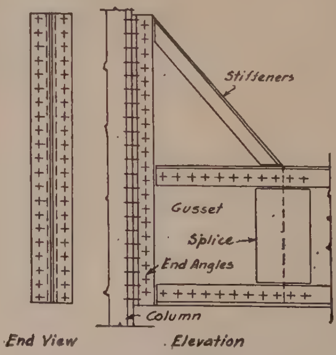


FIG. 10. — GUSSET DESIGN WITH END ANGLES (GUSSET ABOVE)

resist the shear acting at that point, or they may be considered as taking their proportion of the bending moment. The balance of the bending moment is resisted by the strength in the flange angles developed by the rivets connecting the angles to the gusset.

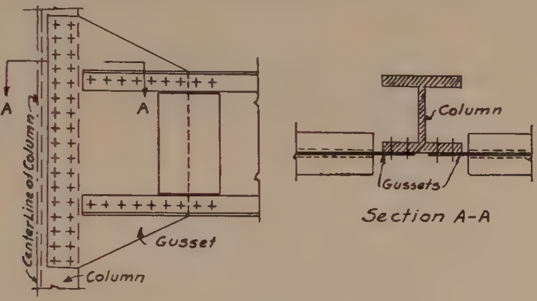


FIG. 11. — GUSSET PLATES RIVETED TO FLANGES OF COLUMN

Gusset Riveted to Face of Column. — In the design shown in Fig. 11 the end angles are omitted and the gusset is riveted directly to the face of the column. The bending moment from the gusset is transmitted to the column by rivets acting in single shear. In other respects the design is the same as in the previous cases.

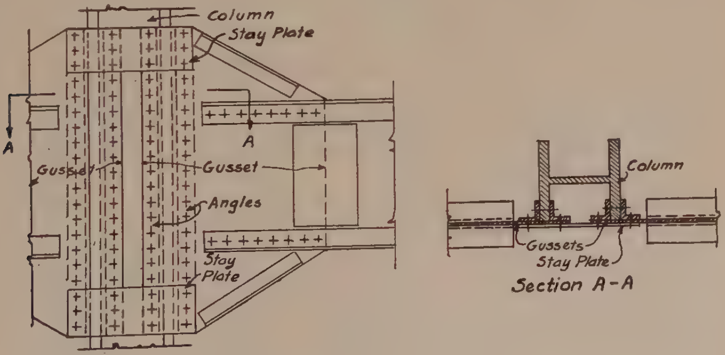


FIG. 12. — END ANGLES CONNECTED TO ENDS OF COLUMN FLANGES

Gusset Connected to Side of Column. — In Fig. 12 the gusset is riveted to two vertical angles connected to the flange of the column. In this case the design of the gusset is the same as in the previous case. The shear transmitted from the gusset to the vertical angles produces in each angle tension in the rivets on one side of the neutral axis, and bearing on the other side of the axis. To stiffen the flanges of the columns against bending, stay plates are provided at the top and at the bottom of the connection.

In Fig. 13 the gusset is riveted to one angle only. To get sufficient strength, a vertical cover plate is riveted to the gusset and to the other vertical angle, thus making the rivets act in double shear. The strength of the connection is the same as if the gusset were riveted to both angles. When the thickness of the gusset is not large enough to develop in bearing the double shear in rivets, it should be increased by a plate riveted to the gusset by one row of rivets.

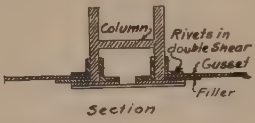


FIG. 13. — VARIANT OF FIG. 12

Gusset Forming Part of Web of Column. — In Fig. 14 a built-up column is used. The gusset forms a part of the web of the column. At the splices in the column two splice plates are used with sufficient rivets to transfer the compression stresses from the web to the gusset, and then back from the gusset to the column web.

This design is used only rarely.

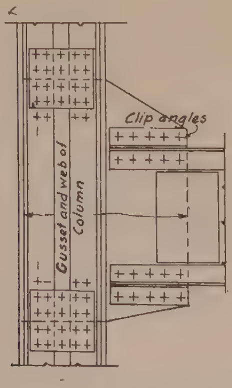


FIG. 14. — GUSSET FORMING WEB OF COLUMN

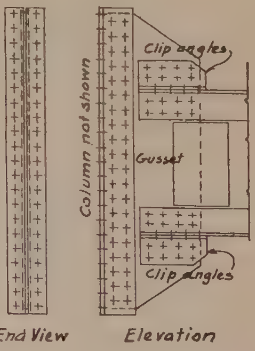


FIG. 15. — GUSSET CONNECTION WITH CLIP ANGLES

Vertical Angles of Unequal Legs. — In the designs shown in Figs. 8 to 10 the gusset must be thick enough so that the bearing of one row of rivets connecting the gusset to the end angles is about equal to the tension value of the two rows of rivets connecting the angles to the column. This often requires comparatively thick gussets. To reduce the thickness of the gusset the vertical angles of unequal legs may be used as shown in Fig. 15, and the longer leg may be riveted to the gusset by two rows of rivets. The bearing of these rivets on the gusset will be appreciably reduced, permitting the use of a thinner gusset.

Use of Clip Angles. — In Figs. 8 to 12 the gusset is long enough to accommodate a sufficient number of rivets in the flange angles to develop the bending moment at the splice. When the available head room does not permit the use of sufficiently long gussets, the required bending moment at the edge of the gusset may be developed by means of clip angles riveted to the gusset and the outstanding legs of the flange angles in the manner shown in Figs. 15 and 16.

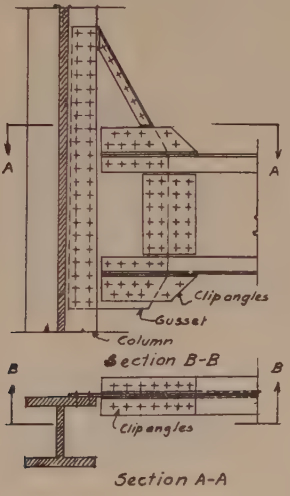


Fig. 16. — Design of Gusset Connection with Clip Angles

In the design shown in Fig. 15, two pairs of clip angles with unequal legs are used. The longer legs are riveted to the gusset by two rows of rivets, and the shorter legs to the flange angles by one row of rivets acting in single shear. To develop the same strength in single shear in the flange angles as is developed by the two rows of rivets by bearing (or double shear) in the gusset, it is often necessary to extend the clip angle beyond the edge of the gusset as shown in Fig. 16.

Gusset Replaced by T-stub. — The gusset and the end angles used in Figs. 11 to 16 may be replaced by a T-stub formed by splitting rolled

I or H sections along their web, as shown in Figs. 17 and 18. The flange of the T-stub takes the place of the angles, and is riveted to the column. By proper selection of the rolled section the thickness of the flange of the T-stub may be much larger than possible to get with the heaviest angle. This is important in connections to column requiring rivets of large diameter. The increased stiffness of the flange reduces the bending in the tension rivets and makes them more effective.

The use of the T-stub is particularly necessary in a design requiring four rows of rivets to connect the construction to the column, as shown in Fig. 18. In this design a heavy H section was used for the T-stub, which has a thickness of flange more than 50 per cent greater than the thickness of the heaviest available angle, and it is therefore about two and one-half times as strong in bending.

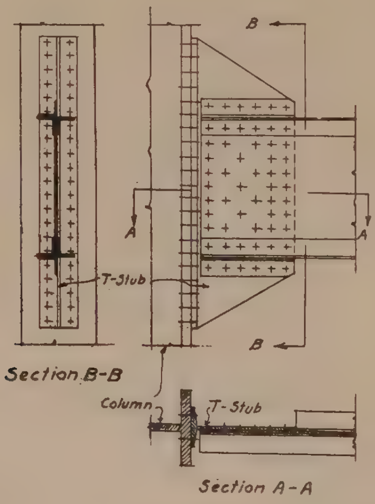


Fig. 17. — Gusset replaced by T-Stub

The connection between the stub and the beam can be made in the same manner as used in the gusset designs. In the design shown in Fig. 17 a different arrangement was used. The T-stub is riveted to the column. The web of the beam overlaps the web of the T-stub, and is field riveted to it. The flange angles on one side of the web also overlap the T-stub and are field riveted to it. Clip angles are also used to develop the flange angles. The bending moment at the edge of the T-stub is equal to the moment developed by the field rivets acting in single shear.

In Fig. 18 the T-stub was made of an H section of the largest available depth. The web of this stub can accommodate only three rows of rivets, which gives only half as many rivets as would be required if the rivets acted in single shear.

To increase the value of the rivets, the web of the beam was placed on one side of the web of the T-stub, and a splice plate was placed on the other side. In this way, by placing the rivets in double shear, their value was doubled. It should be noticed that the same number of rivets must be used in the splice plate on both sides of the splice. Also a row of shop rivets should be used to connect the web to the filler.

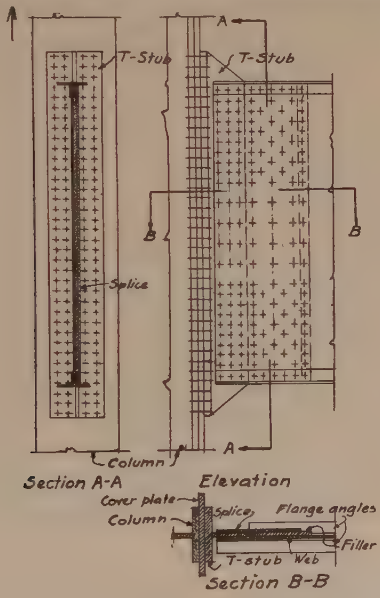


Fig. 18.—Gusset and End Angles replaced by T-Stub

Method of Erection

It is important to decide how the beam will be erected, as this determines which rivets are shop driven and which are to be driven in the field.

A beam shown in Figs. 8 to 10 may be shop riveted complete and then the end angles riveted to the column. In this case all rivets may be figured as shop rivets, with the exception of those connecting the angles to the column.

Where it is difficult to swing a complete beam into place, the end angles may be shipped loose and then field riveted to the column and to the gusset.

In designs shown in Fig. 11 the beam is almost always shop riveted complete with the gussets and then field riveted to the column.

In some designs the gussets or T-stubs may be shop riveted to the columns and the beam field riveted to the gusset. In such cases it is impossible to extend the flange angles on both sides along the gusset. One of the angles is lapped and the other must stop short at the edge of the gusset.

Design of Girder with Gussets

The girder with gussets consists of the girder section between the splices and the gusset sections.

The girder section is designed for the maximum bending moment and maximum shear acting in this section. For girders carrying heavy vertical loads, and for cases in which the wind pressure is comparatively small, the design of the girder section is governed by the positive bending moment. The negative bending moment in the girder section at the splice is then smaller than the maximum positive bending moment. For girders carrying small vertical loads and subjected to heavy wind pressure the negative bending moment at the splice may be larger than the maximum positive bending moment, and therefore it may govern the design. Such a case is shown in the bending moment diagram in Fig. 7.

The depth of the girder is often made larger than would be required by the bending moment.

The design of the gusset connection to the girder is simple. The splice plates may be designed to resist the shear acting at the splice, for which usually one row of rivets on each side is sufficient. Some designers make the splice strong enough to develop the shearing value of the web of the girder. In such designs two rows of rivets on each side of the splice are required.

The bending moment at the splice is resisted by the strength in the flange angles of the girders developed by the rivets connecting the angles to the gusset. The rivets should be figured at their maximum value in bearing (or double shear) when two angles, one on each side, are riveted to the gusset, or in single shear when one angle only is riveted to the gusset. The allowable unit stress specified for wind stresses should be used. The resisting moment equals the value of the rivets times the distance between the gauge lines of the upper and lower angles.

When clip angles are used, the resistance of the rivets to shear in the outstanding legs should be taken as a unit, and in the lower line of rivets it should be reduced according to a straight line with the center of web as a zero point.

It is obvious that the clip angles must be riveted to the gusset with a sufficient number of rivets to develop by bearing (or double shear) the strength developed by the clip angles in the flange angles. In both cases the rivets should be figured at their maximum value.

When two rows of rivets are used in the web splice their value in resisting bending moment may be added to that of the flange angles.

Design of Connection of Gusset to Column

Design with End Angles. — In the design shown in Figs. 8 to 10 the bending moment at the edge of the column is transferred from the gusset to the column by means of the rivets connecting the vertical angles to the column. The bending moment produces in the connection tension and compression stresses in the same manner as in a beam subjected to bending. The intensity of the stresses is zero at the neutral axis, and increases according to a straight line to a maximum in the extreme fibers. In this case in the tensile zone the place of the tensile fibers is taken by rivets acting in tension. (See Fig. 19.) In the compression zone the compression stresses are replaced by the bearing of the angles against the column. The bearing stresses are always small, and the strength of the connection depends upon the resistance of the rivets to tension. To simplify the computation an assumption, erring on the safe side, is made that the compression stresses are also resisted by rivets, and that the compression stresses in the rivets vary in the same manner as the tensile stresses. The neutral axis then will pass through

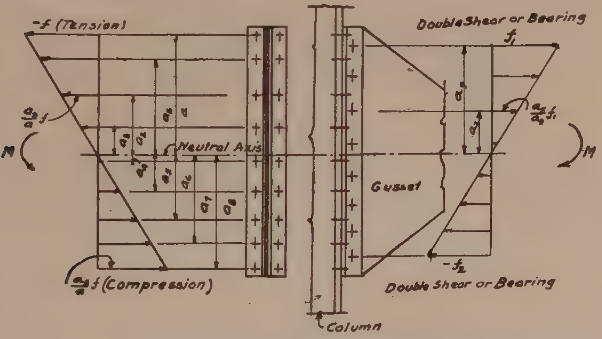


FIG. 19. — STRESSES IN RIVETS DUE TO BENDING MOMENT

the center of gravity of the group of rivets. When the rivets are symmetrically arranged about a horizontal axis, the neutral axis coincides with the axis of symmetry. When the rivets are not symmetrically arranged, the distance of the neutral axis from the bottom line of rivets may be found by adding the distances of all the rivets from the bottom and dividing the sum by the number of rivets. The maximum stress will act in the rivets farthest from the neutral axis. In Fig. 19 the maximum stress resisted by one rivet, f, acts in the top line of rivets. Plotting this value and connecting it with the neutral axis, we get the triangle of stresses. The stresses resisted by any intermediate rivet may be obtained by scaling from the stress diagram. Analytically the stress f_x resisted by an intermediate rivet may be expressed in terms of the maximum stress f by the formula —

$$f_x = \frac{a_x}{a} f$$

in which a_x = the distance of the rivet from neutral axis
and a = the distance from neutral axis to the extreme line of rivets

Using this equation, the stresses in the rivets shown in Fig. 19 are $\frac{a_1}{a} f$, $\frac{a_2}{a} f$, $\frac{a_3}{a} f$, $\frac{a_4}{a} f$ . . . and $\frac{a_8}{a} f$. The sum of tensile stresses of one row is $f\left(1+\frac{a_1}{a}+\frac{a_2}{a}+\frac{a_3}{a}\right)$, and of the compression stresses $f\left(\frac{a_4}{a}+\frac{a_5}{a}+\frac{a_6}{a}+\frac{a_7}{a}+\frac{a_8}{a}\right)$. The sum of tensile stresses must be equal to the sum of compression stresses.

The bending moment resisted by any rivet equals the stress in that rivet multiplied by its distance from the neutral axis. Thus the rivet at a distance a_x resists a bending moment $a_x \frac{a_x}{a} f = \frac{a_x^2}{a} f$. The bending moment resisted by all the rivets in one row is equal to the sum of bending moment of the individual rivets in that row. Thus in Fig. 19 the bending moment resisted by one row of rivets in the connection is —

$$M = f\frac{a^2}{a} + f\frac{a_1^2}{a} + f\frac{a_2^2}{a} + f\frac{a_3^2}{a} + \ldots\ldots f\frac{a_8^2}{a} \quad \text{also}$$

$$M = \frac{f}{a}\left(a^2 + a_1^2 + a_2^2 + a_3^2 + \ldots\ldots a_8^2\right)$$

To get the bending moment resisted by several rows of rivets multiply the above bending moment by the number of rows. The term in parentheses is the moment of inertia of the row of rivets. Therefore —

$$M = \frac{I}{a} f \text{ for one row of rivets} \qquad (1)$$

$$M = m \frac{I}{a} f \text{ for } m \text{ rows of rivets} \qquad (2)$$

where I—moment of inertia of one row of rivets
a—distance neutral axis to the farthest line of rivets
f = maximum stress resisted by one rivet

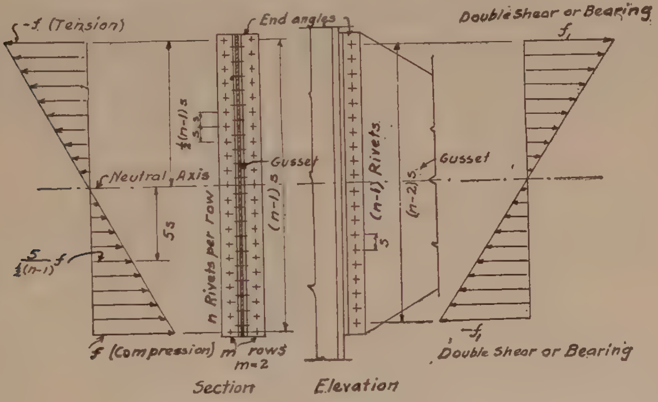


FIG. 20. — RIVETS WITH EQUAL SPACING

Connection with Equal Spacing of Rivets. (See Fig. 20.) — When the vertical spacing of rivets in a row is equal to s the neutral axis passes through the center of the row of rivets.

Let

n = number of rivets in one row
m = number of rows of rivets
s = spacing of rivets
I = moment of inertia of rivets
M = moment of resistance of rivets
C = constant

Distance neutral axis from extreme rivet:

$$d_1 = \tfrac{1}{2}(n-1)\,s \qquad (3)$$

Moment of inertia:

$$I = 2\,m\left[1 + 2^2 + 3^2 + 4^2 + \quad . \quad . \quad . \left(\frac{n-1}{2}\right)^2\right] s^2 \text{ when } n \text{ is odd}$$

$$I = 2\,m\left[0.5^2 + 1.5^2 + 2.5^2 + \quad . \quad . \quad . \left(\frac{n-1}{2}\right)^2\right] s^2 \text{ when } n \text{ is even}$$

Moment of resistance of rivets:

$$M=\frac{f}{\frac{1}{2}(n-1)\,s}\,I=m\,s\,C f \tag{4}$$

where $$C=\frac{4}{n-1}\left[1^2+2^2+3^2+4^2+\;.\quad .\;\left(\frac{n-1}{2}\right)^2\right] \text{ for odd } n \tag{5}$$

and $$C=\frac{4}{n-1}\left[0.5^2+1.5^2+2.5^2+3.5^2+\;\left(\frac{n-1}{2}\right)^2\right] \text{ for even } n \tag{6}$$

The values of C for different values of n are given in Table 1.

TABLE 1.—CONSTANTS C IN FORMULAS (7) AND (8) FOR NUMBER OF RIVETS

NUMBER OF RIVETS n	Constant C	NUMBER OF RIVETS n	Constant C	NUMBER OF RIVETS n	Constant C
3	2.0	17	51.0	31	165.0
5	5.0	19	63.0	33	187.0
7	9.33	21	77.0	35	210.0
9	15.0	23	92.0	37	234.3
11	22.0	25	108.3	39	382.0
13	30.3	27	126.0	41	531.2
15	40.0	29	145.0		

Maximum stress in one rivet:

$$f=\frac{M}{C\,m\,s} \tag{7}$$

The use of the table is very simple.

To compute the number of rivets in one row required to resist the bending moment M with maximum stress in one rivet f, with number of rows, m, and spacing s, find

$$C=\frac{M}{m\,s\,f} \tag{8}$$

Locate this value in the table. The corresponding value of n gives the number of the rivets in one row.

To find the stress f, when M, n, m and s are known, find from table the constant C corresponding to the number of rivets in a row n. Then find f from formula (7).

Use of Table for Unequal Spacing of Rivets.—In some designs the equal spacing of rivets is interrupted in one or two places where the gauge is increased. In such cases the formulas previously given and the table would give results erring on the safe side. Usually the number of rivets in a row found from the table would be one or two rivets larger than necessary.

Rivets Connecting End Angles to Gusset. — The rivets connecting the gusset to the end angles when subjected to bending moment act in double shear or bearing. Above the neutral axis the shear acts in one direction, and below the neutral axis acts in the opposite direction, as shown in Fig. 19. The intensity of the shearing stresses varies from zero at the neutral axis to a maximum in the extreme line of rivets. The neutral axis is found in the same manner as explained in connection with the tension rivets. Formulas (1) and (2) may be used to determine the number of rivets for a given bending moment, or the stress in rivets for known number and arrangement of rivets. For equal spacing of rivets use formulas (4) to (8).

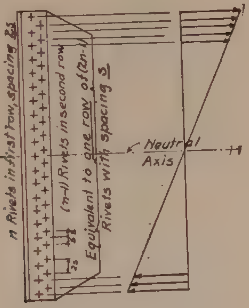

n Rivets in first row, spacing 2s
(n−1) Rivets in second row
Equivalent to one row of (2n−1)
Rivets with spacing s
2s
2s
Neutral
Axis
H
H

Fig. 21. — Staggered Rivets in End Angles

In the end angles usually the number of the tension rivets is found first. The rivets connecting the gusset are then arranged in one row, using the same spacing and staggering them in respect to the tension rivets as shown in Fig. 19. In this way the number of the rivets is obtained automatically. It is then necessary to find the bearing stresses in the rivets from formula (7). The thickness of the gusset should be checked to see whether it is sufficient to develop the computed stress in the rivets by bearing.

It often happens that the required thickness of the gusset to develop the stress in the rivets by bearing is too large, or even that the stresses exceed double shear in the rivets. In such cases unequal angles are used for end angles, with the longer leg riveted to the gusset by two rows of rivets. When the rivets are staggered, as in Fig. 21, they may be considered in computation as one row, with spacing equal to the pitch of the staggered rivets. The stresses may be computed by formula (8), using Table 1.

Gusset Riveted to Face of Column. — When the gusset is riveted directly to the column by two rows of rivets, as shown in Fig. 11, the bending moment produces single shear in the rivets varying in intensity from zero at the neutral axis to a maximum in the extreme line of rivets. As before, the shear acts in one direction above the neutral axis, and in the opposite direction below the axis. The number of rivets may be found from formula (8), using Table 1, in which f is the value of one field rivet in single shear.

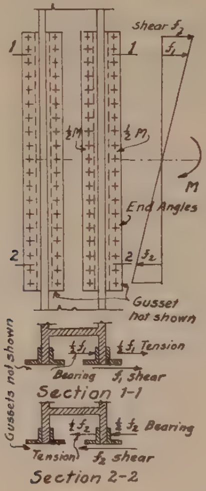


FIG. 22. — END ANGLES ATTACHED TO SIDES OF COLUMN FLANGES

Gusset Riveted to Angles as in Fig. 12. — When the gusset is connected to angles riveted to the legs of the column, as in Fig. 12, the rivets in the gusset act the same as when connected to the face of the column. The shear transmitted by the gusset to the angles produces tension in the rivets connecting the angle to the column above (or below) the neutral axis, and bearing stresses on the other side of the neutral axis (see Fig. 22). When the number of rivets connecting the angles to the gusset is the same in both angles, each angle carries one-half of the bending moment, and the rivets connecting the angles to the column should be computed for one-half of the bending moment. Formulas (4) to (8) may be used in which f is the strength of one rivet in tension and M is one-half the bending moment.

Size of Rivets. — For small bending moments ¾-inch rivets are used in the wind bracing. For larger bending moments the diameter of the rivets is increased to ⅞ inch, 1 inch and even larger. Often smaller rivets are used for shop-driven work, and the diameter is increased for field rivets. The diameter of tension rivets also is often made larger than of the shear rivets, especially where the number of rivets is limited, as in the T-stub designs.

In the design shown in Fig. 3, for instance, the T-stubs are often connected to the column by 1-inch tension rivets, while ⅞-inch rivets are used to connect the webs of the T-stubs to the flanges of the beam.

Tension Value of Rivets. — In a number of connections described in this paper the strength depends upon rivets acting in tension. The specifications are usually silent as to the value of tension rivets. Some specifications even forbid the use of rivets in tension. Several tests made on rivets subjected to tension prove that they are able to carry considerable stresses. The actual value of the rivet depends upon the eccentricity of the application of the stress. The resistance of the rivet to tension for the same eccentricity increases with the increase in stiffness of the leg of the angle or T-stub transmitting the stress.

Until the subject of tensile stresses in rivets is clarified, it seems to be a safe practice to use the same stress in rivets in tension as is used in single shear.

Thickness of End Angles

In the gusset connection shown in Figs. 8 to 10 the bending stresses in the tensile zone are transmitted to the column by tension in the rivets connecting the angles to the column. The forces in the gusset and the resisting stresses in the rivets are shown in Fig. 23. Since the forces and the resisting reactions are not on a line, the outstanding legs of the angles are subjected to bending moments. In resisting the bending moment each outstanding leg acts as a cantilever. The maximum effective bending moment acts at a point where the fillet of the angle begins to be effective, and is equal to the stress in the tensile rivet multiplied by the distance of the center of the rivet from the face of the angle minus 0.25 inch; *i.e.*, the gage line minus the thickness of the angle minus 0.25 inch. The length of the angle resisting this bending moment equals the spacing of the tensile rivets.

Since the tensile stresses in the rivets are not constant, the bending moment in the leg of the angle is not constant but decreases towards the center. It would be too conservative to base the thickness of the angle on the maximum tensile stress in the rivet, because the lower portions of the angle undoubtedly assist the part of the angle with the heaviest bending moment in resisting the stresses. It seems to be permissible to take a section four or five rivets long, compute the total force F acting upon it, base the bending moment on these stresses, and consider this bending moment as resisted by the total length of the section. In Fig. 23 the force is $F=2\ (f_1+f_4)$, where f_1 and f_4 are bearing stresses on the first and the fourth rivet, respectively.

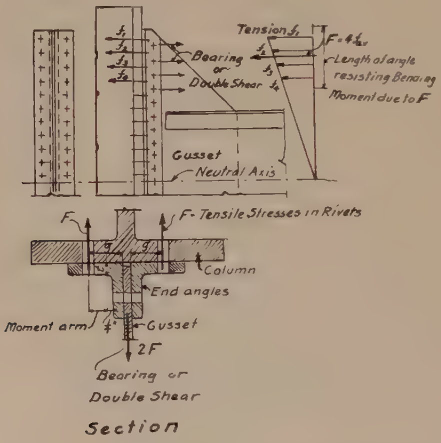


FIG. 23. — BENDING IN END ANGLES

If

F = total force in pounds acting on the section under consideration
l = length of section, in inches
g = gage line, in inches
t = thickness of angle, in inches
f = allowable stress in steel, in pounds per square inch

Then the bending moment, in inch pounds —

$$M=F\ (g-t-0.25) \qquad (9)$$

Thickness of angle:

$$t=\sqrt{\frac{6\ M}{f\ l}} \qquad (10)$$

In computing the bending moment the value of t must be assumed and then checked after the thickness is computed. If there is a large discrepancy between the assumed t and the actual value, it should be recomputed.

The idea of the required thickness of the angle for different average tensile stresses in the rivet may be had from the following table:

THICKNESS OF END ANGLES

	$f_{av.}$ Pounds per Square Inch						
Average stress in rivet .	4,000	5,000	6,000	7,000	8,000	9,000	10,000
Thickness of angle (in inches, outside of fillet)	0.85	0.91	0.97	1.03	1.10	1.14	1.20

Based on a gage line 2.5 inches; spacing of rivets at 3 inches; and $f=16{,}000$ pounds per square inch.

To use this table the stresses in the rivets must be reduced to the basis of unit stresses for vertical loads only, because the unit stress in steel used in computation was $f=16{,}000$ pounds per square inch.

The required thickness of the angle increases with the increase of the gage line. Therefore the value of g should be as small as possible; also, where possible, one line of rivets should be used in one angle. Double line of rivets requires a thickness of angle much in excess of the thickest available angle, even with the ¾-inch rivets. Where two lines of rivets are required, T-stubs must be used in place of the gussets and end angles.

The importance of proper thickness of end angles cannot be overemphasized. The most common mistake made in the design of wind bracing is that the thickness of angles is made much smaller than required. This mistake is made even by authors of textbooks. In several textbooks a rule is given recommending the use of angles ⅝ inch in thickness for a gage of 2½ inches and 1 inch in thickness for a gage of 4 inches without even specifying the size of the rivet or the stress in the rivet.

The effect of insufficient thickness of the end angles is distortion of the angles and also excessive bending in the tensile rivets. To have the stress in the rivets as central as possible it is important to have the outstanding leg of the angle as rigid as possible.

The thickness of the end angles used in Fig. 12 must be designed in the same manner as for ordinary end angles.

Design of the Gusset

Shape of Gusset. — The shape of the gusset depends upon the allowable clearance, which determines whether the gusset should extend above or below the beam, or both. The depth of the gusset depends upon the number of rivets required in the connection to the column. The length of the gusset is governed by the number of rivets in the flange angle of the girder overlapping the gusset required to develop the required bending moment in the girder at the edge of the gusset. If this length is larger than allowed by the clearance, the clip angle design may be used.

Thickness of Gusset. — The thickness of the gusset may be governed by the magnitude of the bearing stresses to be developed by the rivets connecting the end angles to the gusset, or by the bending moment.

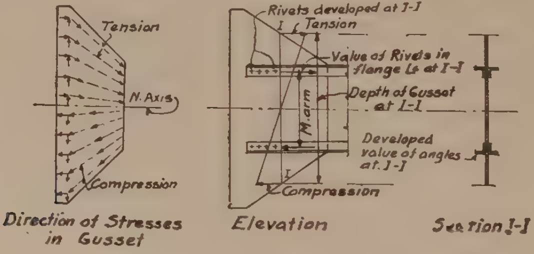


FIG. 24. — STRESSES IN GUSSET

Outside of the connecting angles, as far as the bending moments are concerned, the gusset is a rectangular beam, and the horizontal fiber stresses can be found from the beam formula —

$$f=\frac{6\,M}{t\,d^2}$$

where t = the thickness of the gusset
and d = the depth, measured on a vertical line

When the edges of the gusset are horizontal, the maximum stress in the gusset is horizontal. In the portion of the gusset with inclined edges, the stresses at the edges are parallel to them.

Considering the gusset alone, it is logical to assume that the lines of stresses would be located somewhat as shown in Fig. 24. The flange angles riveted to the gusset alter the lines to some extent. Exact formulas for stresses in the gusset would be too complicated. It is accurate enough to assume that, where the gusset acts alone at any vertical section through the inclined part of the gusset, the bending moment is resisted by the horizontal components of the inclined stresses. The maximum fiber stress at the edges is then equal to the horizontal stress in the extreme fiber found from the formula just given, multiplied by the secant of the inclination of the edge with the horizontal.

For safe design the maximum fiber stresses, whether they act horizontally or at an angle, must not exceed the allowable working fiber stress.

Having determined the thickness of the gusset at the column, it is necessary to investigate the condition at intermediate sections. Consider a vertical section midway between the column and the splice; *i.e.*, section *I–I* in Fig. 24. This section cuts not only the gusset, but also the flange angles overlapping the gusset, the maximum value of which in tension and compression is equal to the stress developed in the angles by the rivets to the left of the section. The moment of resistance of the gusset at section *I–I* plus the moment of the stresses developed in the flange angles may be sufficient to resist the bending moment at that section. If they are not, stiffener angles must be used as explained below.

The stresses acting at section *I–I* may be analyzed more logically by considering the cross section as a composite section, consisting of the plate and the angles. By finding the neutral axis of the composite section and its moment of inertia it is possible to find the horizontal stresses in the extreme fibers. In a gusset which is symmetrical about a horizontal axis the stresses in the upper and lower angles are developed in the same ratio. In a design where the gusset extends only above or below the girder, one set of the flange angles is placed near the neutral axis. The stresses in these are developed slower than in the other set of the angles. The stresses resisted by the angles in this composite section must then be developed by rivets connecting the angles to the gusset.

Stiffener Angles. — When the length of the edge of the gusset, measured from the rivet line in the end angles to the rivet line in the flange angles, is more than thirty times the thickness of the gusset, it should be strengthened by stiffener angles. When the stresses in the gusset computed as explained in the preceding paragraphs are satisfactory, 3″×3″ or 3½″×3½″ angles are sufficient to stiffen the gusset. When, however, the stiffeners are counted upon to assist in resisting stresses, they should be designed for the stress they are figured to resist. In such case compute the bending moment resisted by the gusset and the flange angles, and deduct it from the total bending moment. The difference will be resisted by the stiffeners.

Some textbooks recommend that in computing the area of the stiffeners only the stiffeners and the strips of the gusset touching the stiffeners should be considered as resisting the total bending moment. This disregards the effect of the rest of the gusset and of the flange angles riveted to the gusset. The stiffeners obtained by this method are too heavy, and a good deal of the material is wasted. Also this method requires a much larger number of rivets than necessary.

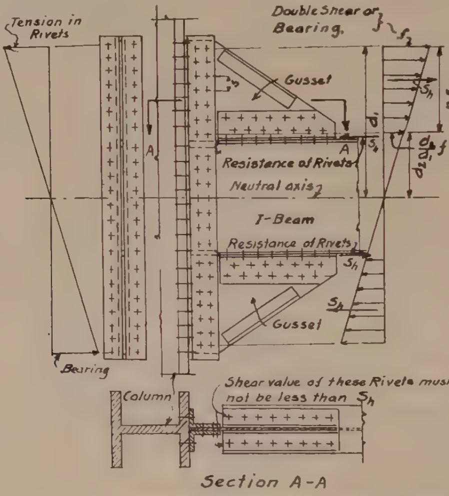


FIG. 25. — GUSSET CONNECTION FOR I-BEAM

Force S_h acting between I-beam and gusset

3. GUSSET CONNECTIONS FOR ROLLED SECTIONS

Rolled beams used as horizontal members in a wind-resisting frame may also be connected to the column by means of gussets. The details of the connections differ to some extent from those used in built-up girders. The most commonly used designs are shown in Figs. 25 to 32.

End Angles Riveted to Beam and Gusset. — In the design shown in Fig. 25 two triangular gussets are used, one above and one below the beam. The end angles are riveted to the gussets and also to the web of the beam. To avoid using fillers, the gusset is made of the same thickness as that of the web of the beam.

The number of rivets connecting the angles to the column and to the beam and gussets, respectively, is governed by the bending moment at the edge of the column. The rivets are equally spaced, with the exception of the larger spacings required at the juncture of the beam and the gussets. The number of rivets may be found from formula (8), using Table 1, as for equal spacing of rivets. The result would be somewhat in excess of the required number because of the two increased spaces. The excess is not likely to amount to more than one line of rivets. If desired, the results could be recomputed, using formula (2).

The gusset plate and the web of the beam are usually thin and not sufficient to develop high bearing stresses in rivets. Therefore the end angles are always of unequal legs, with the longer legs riveted to the gusset by two rows of rivets. The stresses in the rivets may be found by formula (8), using Table 1. The results would be somewhat larger than the actual stresses.

The thickness of the end angles should be determined in the same manner as explained in connection with built-up girders.

The gusset is attached to the beam by means of horizontal angles which are riveted to the gusset and to the flanges of the beam. The number of rivets connecting the flanges to the angles depends upon the magnitude of the moment of resistance to be developed in the beam at the end of the gussets. This moment equals the value of the web of the beam in bending plus the value in single shear of the rivets connecting the flange to the horizontal rivets multiplied by the depth of the beam. It is obvious that it is useless to develop by means of rivets a larger moment than the moment of resistance of the net section of the beam.

The minimum number of rivets required to connect the horizontal angles to the flange of the beam is governed by the bending moment at the edge of the column. By referring to Fig. 25 it is obvious that the horizontal shear acting at the flange level is represented by the trapezoid of bearing forces produced in the end angles above the flange of the beam. They must be resisted at the flange level by the rivets connecting the horizontal angles to the flange. If the number of rivets is not sufficient, the connection would not remain intact, but would shear off at the flange level. The magnitude of these forces is —

$$S_h = \tfrac{1}{2}\,(n_1+1)\left(1+\frac{d_2}{d_1}\right) m f \qquad (11)$$

where n_1 = number of rivet spacings in a row above the flange of beam

d_1 = distance from the neutral axis to the extreme row of rivets

d_2 = distance from the neutral axis to the line of rivets just above the flange

f = stress of rivet in bearing

m = number of rows of rivets

This formula may be used also when the flange is below the neutral axis of the rivets in the end angles. The value d_2 is then negative. The minimum number of rivets required to connect the horizontal angles to the flange equals S_h divided by the value of one rivet in single shear.

The number of rivets connecting the horizontal angles to the gusset must be large enough to develop in bearing on the gusset the same strength as is developed by the rivets in the flange of the beam by single shear.

When the gusset is placed only below or above the beam the free flange is extended to the column and riveted to it by means of clip angles. In Fig. 29 a T-stub was used to connect the top flange to the column. The stresses resisted by the T-stub are the same as explained in connection with Fig. 4. The relation between the tension rivets in the column and the shear rivets at the beam is also the same. In computing the moment of resistance of the rivets at the column the stress resisted by the stub should be represented by a proper number of rivets placed at the center of the stub. Then the neutral axis and the moment of inertia of the whole group is found, always keeping in mind the concentrated number of rivets at the stub.

Gusset and Web Connected directly to Column. — In Fig. 28 the gusset and the web are connected directly to the column. The rivets in the column act in single shear. Their number may be found by formula (8) and Table 1 with the same exactness as explained in connection with end angle design. In other respects the design does not differ from those described before.

Gusset and Web Connected to Side of Column. — In Fig. 27 the gusset and the web of the beam are riveted to vertical angles riveted to the flanges of the columns. This design corresponds to that shown in Fig. 12 for built-up girders. Attention is called to the clip angle shown in section B-B.

Gusset Replaced by T-Stub. — In Fig. 30 the gusset and the horizontal angles are replaced by a T-stub formed by splitting an I-beam through the web. The number of rivets connecting the stub to the column and to the beam are computed in the same manner as for regular gusset designs.

In Fig. 31 the gusset and the end angles are replaced by a T-stub. The beam is riveted to the web of the T-stub. Clip angles are used to develop the flange of the I-beam.

End Angles Riveted to Gussets only and not to Web of Beam. — In the design shown in Fig. 26 the gussets only are riveted to the column.

The vertical connecting angles consist of two sets of short connecting angles.

The number of rivets required in the connecting angles to develop the bending moment at the edge of the column may be found by using the formulas below.

As evident from Fig. 26 the total tension in the rivets equals —

$$T = nf\frac{1+(n-1)\frac{s}{d}}{1+2\,(n-1)\frac{s}{d}}m \qquad (12)$$

and the bending moment

$$M = nfd\left[1+\frac{4\,(n-1)^2\left(\frac{s}{d}\right)^2}{3\left(1+2\,(n-1)\frac{s}{d}\right)}\right]m \qquad (13)$$

where T = total tension in rivets
n = number of rivets in one angle
m = number of rows of rivets
s = spacing of rivets
d = distance between the lowest top rivet and highest bottom rivet
f = value of rivet in tension

Formula (13) may be written —

$$\frac{M}{mfd} = n\left[1+\frac{4\,(n-1)^2\left(\frac{s}{d}\right)^2}{3\left(1+2\,(n-1)\frac{s}{d}\right)}\right] = C_2 \qquad (14)$$

The values of C_2 for various $\frac{s}{d}$ and various values of n may be computed and a diagram prepared.

With the aid of this diagram it is easy to find the number of rivets for known values of $\frac{M}{mfd}$ and $\frac{s}{d}$.

When the number of rivets is known it is easy to compute either the stress or the resisting moment.

Discussion

WALDO F. PIKE:* As Mr. Smulski has said in his paper, many building codes and specifications make no direct reference to the subject of rivets in tension, except perhaps to say that such rivets shall be avoided. The Boston Building Law makes no reference to them.

Tests show that rivets have considerable value in tension. The failures usually occur in the shank of the rivet. It might be expected that the heads would fail by shearing, but this seldom happens.

There is some disagreement between different authorities as to the proper allowable working value, consequently a few references to these may prove of interest.

(1) The latest "Standard Specification for Structural Steel for Buildings," as adopted by the American Institute of Steel Construction, says, for rivets in tension: "On area of nominal diameter of rivet, 13,500 pounds per square inch."

(2) Ketchum, in his "General Specification for Steel Frame Buildings," says: "Rivets shall not be used in direct tension, except for lateral bracing where unavoidable; in which case the value for direct tension on the rivet shall be taken the same as for single shear."

(3) Burt, in his book on "Steel Construction," says: "The unit stress allowable is the same as for shear."

(4) Hool and Kinne, in "Structural Members and Connections," quote Mr. Fleming: "He recommends a unit stress of 7,000 pounds per square inch for rivets in tension."

(5) The St. Louis building code (1927) says: "The tensile stress in pounds per square inch net section for rivets shall not exceed 9,000 pounds."

(6) Mr. C. R. Young, in a paper presented before the American Institute of Steel Construction in 1927, recommends the following formula, which was derived after a series of tests:

$$P_t = 21{,}000 - 8{,}000\,d - 5{,}500\sqrt{e}$$

in which —

P_t = permissible tensile stress in rivet
d = diameter of rivet in inches, before driving
e = eccentricity of loading on rivet, in inches (e may be taken as about two-thirds the value of the gage for the outstanding leg)

The above formula provides a factor of safety of 4.

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Taking the foregoing recommendations and working out a tensile value for ¾ inch rivets gives the following results, using the recommended American Institute of Steel Construction shear values where they apply:

(1) A. I. S. C.	5,960
(2) Ketchum	5,960
(3) Burt	5,960
(4) Hool and Kinne	3,093
(5) St. Louis Building Law	3,976
(6) Young (assuming 1¾ inch gage)	4,000

For members carrying wind load only, it is customary to allow an increase in permissible working stresses of from 20 to 50 per cent, and the rivet values given above may consequently be increased by a certain percentage.

A value of two-thirds single shear was used in one of the tall buildings recently erected in Boston. An arbitrary value of 5,000 pounds per rivet was allowed on another.

The intention of this discussion is not to recommend any particular value to use for rivets in tension, but simply to show the variation in values now in use. The adoption of single shear as a tensile value by the American Institute of Steel Construction is the latest ruling, and possibly may be made a part of many building codes.

MR. LEWIS E. MOORE * (presiding): It seems that using the single shear value for rivets in tension practically amounts to using three-fourths of the regular tensile stress.

MR. W. W. BIGELOW: † I am very glad that Mr. Smulski has written this paper on the "Design of Wind Bracing" as I think he has given us some valuable ideas and made available a good many practical connection details. Many of the connections he illustrates are admittedly indeterminate, and perhaps the exact analysis of any of them is involved. It seems, however, that the analysis of the simple connection shown in his Fig. 6 should be capable of a straight-forward treatment.

This connection in Fig. 6 illustrates a method of fastening an I-beam to a column by means of an angle on top of the beam and another on the bottom. Mr. Smulski says: "In some textbooks the tensile stresses in the rivets are computed by dividing the bending moment

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at the column by the distance between the top and bottom line of rivets connecting the angles to the column. This is *obviously* incorrect. . . ."

It is *not* obvious to me that this is incorrect. It seems to me that what produces bending at this junction point is the same whether we are considering what takes place in the end of the beam *or* what takes place in the column at the end of the beam. I can't follow Mr. Smulski when he shows a certain moment, M, required at the end of the beam to resist the external moment, and a different moment of $M+2\ M$ required at the face of the column to resist the same external moment.

I believe the proper way to find the tensile stress in the rivets through the column *is* this method which Mr. Smulski says is incorrect. I should divide the external moment by the distance between the rivets to get the stress in the rivets, assuming, with Mr. Smulski for the time being, that the center of compression would be located at the lower rivets.

I visualize this connection as two angles so thoroughly riveted to the I-beam as to become an integral part of it, and the outstanding legs of the angles to be stiff enough to resist the forces to which they are subjected. To carry my picture to the extreme, I imagine the angles becoming very large while the depth of the beam remains unchanged. The arm of the resisting couple at the face of the column has become large and the forces in this resisting couple have become correspondingly small. This is quite the opposite of Mr. Smulski's method, which leads to the conclusion that the stresses in these column rivets do not change, no matter how large the angles may become, so long as the depth of the I-beam is a constant. This is not obvious to me, in fact, I think it is incorrect.

MR. EDWARD A. VARNEY:* In the paper it was stated that the T-sections were formed "by splitting I-beams through the webs." Was this the literal intention? Usually one flange is burned off to make the T-section. If the flange of the T-section is shop riveted to the column, it would seem as if awkward field connections would result. On the other hand, if field driven, less efficiency results. Accurate fabrication is required to make the angles on the beam web function properly. This one-sided connection is used presumably to facilitate erection. It would introduce some eccentricity, however, and a pair of angles might be more desirable.

When a loose filler detail is used between the T-section and the flange of the beam, the amount usually added is 15 per cent increase in the number of rivets for each loose filler.

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When the tensile stress in the stub of the T-section is investigated, it should be borne in mind that the *net* section should be used.

One point which has not been clearly brought out is that of combining the dead and live load moment and the wind moment, for the design of the connections. Should this be a negative moment, due to end restraint for live and dead load plus wind load moment, or just the wind moment? Some variance of opinion seems to exist in regard to this.

Mr. Smulski's discussion has been very helpful on design of connections for wind bracing. The methods of computing bending moments and shears due to wind pressure were presented some time ago in a paper by Mr. Robins Fleming before the Society. This leaves only the part of checking up the members themselves for combined dead, live, and wind stresses. The columns generally are unaffected. The increase in direct load due to wind is easily taken care of by the increased allowable stress. Ample sectional area is available to resist shear due to the wind at the points of contraflexure.

The direct stresses in the floor girders due to wind are comparatively small, and the floor construction itself tends to provide relief. This leaves only the bending in the floor girders due to combined loading. In many cases, the size is satisfactory because of the increased working stresses. However, this feature should be investigated to take care of the exceptions.

The stiffness of brackets and their greater depth cause restraint at the ends of the girders. Negative moment due to dead and live load may result with a decrease in the positive moment. The straight-line variation of wind moment and the moment diagram for dead and live loads for a fixed-end beam may be combined. However, a practical consideration enters. The end connections are usually made alike to provide for wind reversal. The columns at each end are often alike. Hence, equal stiffness at each end results. The previous combined diagram may then be shifted so that the positive moment at the right end has the same value as the negative moment at the left end. The *critical* maximum moment to design for may be the maximum from either case.

This method would be complicated and laborious, and practically the same maximum moment values may be obtained by *assuming* the beam as simply supported. The wind moment and the dead and live load moment can be combined. The object is to get a simple procedure for obtaining the maximum moment and how to handle it, because of the ratios of the two kinds of moment and the fact that the allowable fiber stress is increased.

Fig. A shows a shear diagram for dead and live load (uniformly distributed), one for wind shear, and the combined shear diagram. The zero point of the shear is no longer at mid-span, but moves over some distance "a." Fig. B shows a moment diagram for dead and live load (uniformly distributed), one for wind moment, and the combined moment diagram. The latter is a form of parabola with its maximum ordinate at some point other than mid-span, namely, at the point of zero shear. The maximum moment exceeds the live and dead load moment by some amount, X. The wind moment is some function of the live and dead load moment, say AM. The wind shear is —

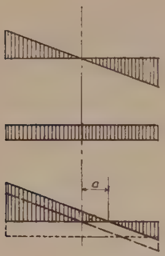

FIG. A

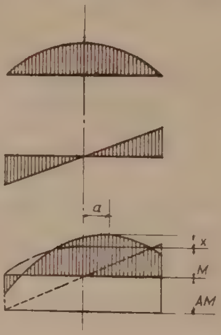

FIG B

$$\text{Wind shear} = \text{Wind moment} \div \frac{L}{2},$$

or —

$$= \frac{AM}{\frac{L}{2}} = \frac{\frac{A(wL^2)}{8}}{\frac{L}{2}} = \frac{AwL}{4}$$

The distance to the point of zero shear from mid-span is —

$$A\frac{wL}{4} \div w = A\frac{L}{4}$$

or from the left hand end is —

$$A\frac{L}{4} + \frac{L}{2} = (A+2)\frac{L}{4}$$

By the properties of the parabola —

$$\frac{X}{X+M+AM}=\frac{A^2\left(\frac{L}{4}\right)^2}{(A+2)^2\left(\frac{L}{4}\right)^2}$$

$$=\frac{A^2}{(A+2)^2}$$

From this —

$$X \doteq M\frac{A^2+A^3}{4A+4}$$

The maximum moment is then —

$$Mmax=M+X$$

$$=M\left(1+\frac{A^2+A^3}{4A+4}\right)$$

This may be simplified as —

$$Mmax=M\left(1+\frac{A^2(A+1)}{4\ (A+1)}\right)$$

or

$$Mmax=M\left(1+\frac{A^2}{4}\right)$$

Say a code allows 30 per cent increase over the usual allowable for combined stress. If X is less than 30 per cent of M, the design of the girder is governed by the live and dead load moment. When $X=0.3\ M$, $A=1.1$. Stated another way, unless the wind moment is 1.0 per cent of the live and dead load moment, the design of the girder is governed by the live and dead load moment, at usual fiber stresses. When A exceeds 1.1, the design is governed by —

$$\frac{10}{13}(X+M) \text{ at usual fiber stresses}$$

Let $K=1+X$. A diagram may be made for values of K corresponding to varying values of A, such as 1.1, 1.2, etc.

The procedure then is simply —

(*a*) Figure live and dead load moment
(*b*) Figure wind moment
(*c*) Get ratio of (*b*) to (*a*), called A
(*d*) If A is less than 1.1, design girder for live and dead load moment, at regular fiber stress
(*e*) If A exceeds 1.1, design girder for KM, at regular fiber stress

If the allowable increase over the usual fiber stress is other than 30 per cent, a new diagram for K would have to be made, and the value of A when live and dead load moment only governs.

For Boston Law, 20 per cent increase in stresses is allowable. Then —

For $X = 1.2\ M$, $A = 0.9$, and

For $A = 0.9$	$K = 1.0$
1.0	1.04
1.1	1.09
1.2	1.13
1.3	1.18
1.4	1.24
1.6	1.37
1.8	1.51
2.0	1.67

While all discussions of wind bracing may be theoretically correct, some practical judgment must be exercised to have the whole arrangement reasonable. Many practical factors enter, such as the transmission of wind laterally by solid floors, the effect of partitions and walls, the fireproofing of the framing, the assumptions that all joints are perfectly rigid, that the points of contraflexure of the columns are at mid-height, that the columns are of the same moment of inertia, that the variation of wind moment is as a straight line, and the various methods assumed of wind distribution through the frame.

PROF. FREDERIC N. WEAVER:* Mr. Smulski's paper has dealt wholly with construction details. There may be some present who are wondering how the moments and shears in the girders and columns are obtained. In addition to Mr. Fleming's paper, which has already been mentioned, I might cite a paper in the "Proceedings of the American Society of Civil Engineers" for May, 1928, by Ross and Morris, in which a 48-story building is analyzed by the slope-deflection method and also by an approximate method which gives very good results.

The members of the Boston Society of Civil Engineers, however, will be interested to know that in October, John Wiley & Sons will publish a book by Professors Hale Sutherland and Harry L. Bowman entitled "Introduction to Structural Theory and Design." In this book Professor Bowman develops an approximate method of his own. From a glance at the page proofs it appears that the method is simple and practicable, and probably more accurate than most approximate methods, in that the points of contraflexure in columns and girders are not assumed to be at the mid-points but in positions corresponding more closely with the actual points of contraflexure.

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Mr. LeRoy M. Hersum:* Referring to Mr. Bigelow's question regarding the stress in rivets in the clip angle on the top of the beam, where Mr. Smulski uses a formula of $M = M_1 + 2\,M_2$, etc., I think he is treating the clip angle as a flexible member with considerable deflection at the heel of the angle. This may result in a toggle-like action about the toe of the upstanding leg of the angle with a larger stress in the tension rivets.

Mr. Lewis E. Moore: It seems to me, with respect to Mr. Hersum's question, that you can take the angles on the top and bottom of the beam and figure the tension as though the moment were based on the distance between the rivet holes, and also as though the moment were based on distance between angles or the depth of the beam. Then draw a free body diagram of the angles, which will show, when all the forces are represented, that the forces exerted between the top and bottom surfaces of the beam and the surfaces of the top and bottom angles form couples which account for the apparent discrepancy which Mr. Hersum mentions.

Mr. S. C. Jemian:† I think many of us were expecting that Mr. Smulski would bring forth a new method of design, or a way of finding the bending moments, axial forces, and shears, in a tall building, due to horizontal forces. Mr. Smulski, in going over his main subject, namely, the design of joints, assuming that the designing engineer has already done so, explains why he has disregarded this point. Nevertheless, the determination of the above factors constitutes a very essential part of wind bracing. Therefore I think that a few words about this phase of the problem will be in harmony with the subject.

Recently I had occasion to look into the design of several buildings with reference to wind and earthquake forces. The method used can be classified under "approximate methods," though entirely different from those recommended by Mr. Fleming. I am not enthusiastic about practical methods in preference to theoretical methods, but when we realize the difficulty of solving the problem accurately, it seems advisable to adopt an approximate method which is not altogether arbitrary.

Before going over to the proposed method, I would like to speak about some interesting experience that I had several years ago. I was given the task of comparing different methods, both theoretical and approximate, in order to choose one to apply to the design of a certain tall building. I considered the "slope and deflection" method, and the method developed by Messrs. Ross and Morris. Neither of them, however, was practicable in our case. The first one, due to the labor involved, and the seond, on account of limitation of the diagrams for values "*K*." The main objection to both methods was that we had to design each bent (column line) as a separate unit carrying its part of the wind load coming from tributary bays, disregarding the relative stiffness of all other bents, which connected together, constituting a "rigid-frame in the space." We know that each bent will carry part of the total wind load in proportion to its relative stiffness, but not the amount assigned to it by the common methods in use. The stiffness of any steel frame depends upon the sections of its members and their lengths. Therefore the distribution of any load has to follow the principle of relative stiffness.

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Several engineers have developed an approximate method which takes into consideration the relative stiffness of columns to a certain degree. They distribute the wind load in proportion to the moments of inertia of the columns, disregarding the effect of the beams on the stiffness of the bent.

The proposed method partially takes care of the effect of the beams, as long as they vary in depth between the different column lines at the same story. The method is applicable to the whole building at the same time, instead of to a column line alone. It is based on the following assumption, which constitutes the basic condition of the "slope and deflection" method and others: any building under the action of horizontal forces will deflect so that all column ends of the same floor will be subjected to the same deflection. Considering rigid connections at the joints and the horizontal movement of the floor without any deformation, each column-half can be considered a vertical cantilever, restrained at floor grades. The deflection of a cantilever ideally restrained and with a single load S, normal to its axis, acting at the free end is expressed:

$$D = \frac{Sl}{3\,EI}$$

This expression is constant for all cantilevers of the same story; therefore:

$$D = \frac{S\,l^3}{3\,EI} = \text{Constant} = C;\ \text{and}$$

$$S = \frac{3\,EI}{l^3}\,C = C_1 \cdot \frac{I}{l^3}\ \text{where}\ C_1 = 3\,EC = \text{Constant}$$

This equation shows that the horizontal shears in the columns are in direct proportion to the moment of inertia of columns and inversely proportional to the third power of the cantilevering lengths of columns. This cantilevering length is equal to the half of the clear height of the column, but not the story height. Thus the use of clear height of column provides a means of introducing the effect of the beam upon the distribution of the horizontal loads between all the columns of the same story, satisfying partially the fact that a bent with deeper beams will carry more load. This effect disappears in case the beams of the same story have the same depth but different values of I. In such case the error can be considered negligible.

When the above factors are determined, we proceed as follows:

1. Locate the points of contraflexure at clear mid-heights of columns.

2. Total wind load, above mid-story under consideration, is to be distributed between all columns by following the formula:

$$S_1 = S \frac{\dfrac{I_1}{\left(\frac{h_1}{2}\right)^3}}{\Sigma \dfrac{I}{\left(\frac{h}{2}\right)^3}}$$

3. Having determined S_1 and S_2 of each column above and below a floor respectively, find the sum of the moments of column-halves above and below the joint; distribute this sum to the adjoining beams in proportion to their values of I and inversely proportional to their clear length l; as the relative stiffness of bending members is in proportion to l of the members. The introduction of clear lengths of beams will take care, in a degree, of the type of joint construction.

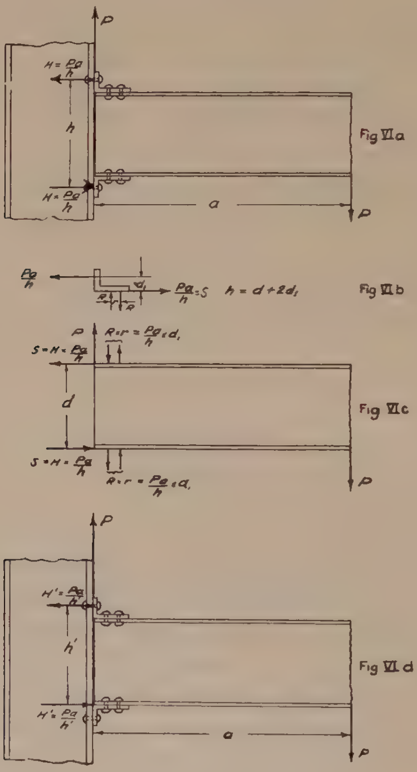
4. Having the moments at each end of a beam, the point of contraflexure of the beam is located and consequently the vertical shear in the beam can readily be determined. Note that this point can be anywhere in the span.

5. Having determined the vertical shears in the beams adjoining a column, the axial load of the column at any story can be obtained by the algebraic sum of all vertical shears acting on the adjoining beams above the section in consideration.

6. The axial loads in the beams are then determined by a simple rule of static equilibrium.

PROF. LEWIS J. JOHNSON:* Mr. Bigelow's main point seems to me well taken and I wish to offer the following in support of that view.

When Mr. Smulski stated that some textbooks are "obviously incorrect," regarding a matter involved in his Fig. 6, he had momentarily lost sight of the two couples brought into action upon the girder flanges (one couple upon each flange) through the hitch angles. He perceives that there are couples acting upon the hitch angles from the girder. But he overlooks that these couples react just as certainly from the hitch angles upon the girder. The tension in the rivets in the vertical leg of the upper hitch angle is actually what he dissentingly reports that some textbooks say. All this will be clear from Figs. VI*a*–VI*d*, for which but a word of explanation need be offered.



FIGS. VI*a*–VI*d*

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Fig. VI*a* shows the two couples Pa and Hh under which the body consisting of the girder and its hitch angles is in equilibrium, whence $H=\frac{Pa}{h}$; and the moment which the connection at the column has to resist is merely Pa, with no increase (or decrease) due to the eccentricity of the connections.

Fig. VI*b* shows the upper hitch angle and the forces (two couples) under which it is in equilibrium, whence (and this is the simplest way to get it) $S=\frac{Pa}{h}$; and also $Rr=P\frac{a}{h}d_1$.

Fig. VI*c* shows the girder properly isolated, with the two couples Rr and the couple Sd brought into action on it by the column through the two hitch angles. From this, setting the sum of these three couples equal to Pa we have $Sd+2\,Rr=Pa$; but, from VI*b*, $Rr=P\frac{a}{h}d_1$ and, from VI*a*–VI*c*, $h=d+2\,d_1$

Substituting $$Sd+\frac{2\,Pa}{d+2\,d_1}d_1=Pa$$

Whence $S=\frac{Pa}{d+2\,d_1}=\frac{Pa}{h}$, the correct value (identical with H) which Mr. Smulski also would have obtained, instead of $S=\frac{M}{d}$, if he had not in this instance ignored the two couples Rr; for M, his "original bending moment," can hardly mean anything other than moment of the couple (which I call Pa) which the girder through its connections brings to bear upon the column. The only value of M which could yield $M=Sd$ is $Pa\frac{d}{h}$; and such a quantity would hardly be called the "original bending moment."

Pa, being a question of nothing but the product of the vertical force P and the horizontal distance a, is wholly independent of the distance apart of the upper or tensile rivets and the center of compression at the lower hitch angle. And of course Hh must equal the fixed Pa, irrespective of the magnitude of h. This means the larger the h, the less the H. It certainly does not mean that, as the paper seems to state, the larger the h, the larger the bending attack upon the column by the shearing force P at the contraflexure point of the girder.

Fig. VI*d* is inserted to indicate how, in general, once the center of compression on the column face is assumed, to get the tension H' in the upper rivets and, what is of the same amount, the shear in the rivets in both flanges of the girder.

From this figure it is seen that

$$H' = \frac{Pa}{h'}$$

in which h' is the distance between the tension rivet and the center of compression wherever the latter may be.

Since, owing to the only moderate stiffness of the angle legs, this center of compression can hardly be so low as the level of the rivets in the vertical leg of the lower hitch angle, it might be wise for simplicity and with error on the side of safety (except in the unlikely case of a direct contact of the girder end with the column face) to make a practice of taking h' as shown in Fig. VId with value $d+d_1$, instead of $d+2\,d_1$ as in Fig. VIa.

I should like to ask Mr. Jemian if he can give an estimate as to how much addition to the work of computation his method is likely to make.

MR. JEMIAN: As to the labor and time, the proposed method involves a little more work than the common methods on account of the accuracy of distribution of horizontal shears. The rest of the operation is as easy as any other practical method.

My latest work in this line was the design of a group of five buildings, separated only with one expansion joint. The earthquake forces and wind loads were carried by concrete walls, the floors acting as horizontal beams to transfer the loads to the walls. It is evident that the concrete framing could not have the simplicity of a steel framing on account of architectural requirements for window and door openings. I was able to investigate the concrete skeleton in both directions and reinforce it fully in a period of time of two and one half weeks. For relative stiffness of columns and beams I used full concrete section, disregarding the reinforcement.

In conclusion, I can state that the proposed method does not involve much more labor than the several so-called "workable" methods, and I believe it approaches the theoretical condition to a greater degree of accuracy.

MR. SMULSKI: From Mr. Pike's interesting discussion, it is evident that additional studies and tests are necessary to determine the strength of rivets in tension. This has been alluded to in my paper. In making tests it is important to use angles of the same rigidity as used in practice. In some tests thus far made the angles were not rigid enough and as a consequence excessive bending was produced in the rivets.

Mr. Bigelow and Professor Johnson are right in their statement that the suggested method of computing stresses in rivets in channel stub connections, as given on page 500, is not correct. As Professor Johnson remarks, I have omitted the effect of the vertical couples, which he calls Rr, which led to erroneous results. I endorse the formula suggested by him —

$$S=T=\frac{M}{h'}$$

where S = shear in rivets connecting beam to angles
T = tension in rivets connecting angle to column
M = combined bending moment at the column due to vertical loads and wind pressure
h' = distance bottom of beam to center line of tension rivets.

Mr. Varney questions the statement that T-sections are formed "by splitting I-beams through the webs." The word "splitting" was used so as to avoid specifying the means by which it is accomplished. Not long ago a number of T-sections formed by burning off one flange failed. This aroused some doubts in the safety of this method. Tests made after the paper was written seem to prove that burning off the flange does not injure the metal in the T-section.

The method of combining bending moments for vertical loadings with those for wind pressure, suggested by me, are shown in Figs. 2*a* and 7. In my opinion the beam should always be considered as restrained when bending moments for vertical loadings are computed, as the negative bending moments at the columns increase appreciably the total bending moments for which the connection must be designed.

Mr. Jemian regrets the fact that no mention has been made in the paper of the method of computing the bending moments for the wind pressure. The paper as originally prepared included two sections, one of which dealt with the design of connections and the other with a novel method of computing bending moments with proper consideration of the variation in the relative stiffness of the various members. It was found that the length of the paper was excessive. It was, therefore, divided into two papers, of which this is the first.

OF GENERAL INTEREST

AMERICAN SOCIETY OF CIVIL ENGINEERS AWARDS THE JAMES LAURIE PRIZE FOR 1930 FOR PAPER ON "THE O'SHAUGHNESSY DAM AND RESERVOIR."

The James Laurie Prize of the American Society of Civil Engineers, for the year 1930, has just been awarded to John H. Gregory, C. B. Hoover and C. B. Cornell, members of that Society, for their paper entitled "The O'Shaughnessy Dam and Reservoir." Mr. Gregory, a member also of the Boston Society of Civil Engineers, is a member of the faculty of the Engineering School of The Johns Hopkins University, and is serving the Public Improvement Commission of the City of Baltimore as consulting engineer on the design and construction of the Prettyboy Dam; Mr. Hoover is superintendent of the Water Works of Columbus, Ohio; and Mr. Cornell is the engineer recently employed by the Public Improvement Commission of Baltimore to take charge of the construction of the Prettyboy Dam.

Each year the American Society of Civil Engineers, the oldest national society of engineers in the United States, awards two prizes for the two best papers printed in the transactions of the Society for the year immediately preceding, and describing in detail completed works of construction and their cost. The prizes are known as the Thomas Fitch Rowland Prize and the James Laurie Prize. The first of these prizes was endowed in 1882 by the late Thomas Fitch Rowland, a well-known civil engineer and Honorary Member of the Society. The second prize was established by the Society in 1912 in honor of the late James Laurie, first President of the Society, and holding that office for fifteen years, from 1852 to 1867. Each prize consists of an honorarium and an engraved certificate. The prizes are presented at the annual meeting of the Society in New York in January.

With the award of the James Laurie Prize, just made, Mr. Gregory will have received both prizes, he having been awarded the Thomas Fitch Rowland Prize in 1910 for his paper entitled "The Improved Water and Sewage Works of Columbus, Ohio;" and it is of interest to note that in this paper, among other things, Mr. Gregory describes the design and construction of the Julian Griggs Dam at Columbus, the first of the two dams built by the city; and further, that both papers deal with completed works of construction for the city of Columbus.

The O'Shaughnessy Dam was designed and built by the Bureau of Water Works Extension of Columbus, and was completed in 1925. Mr. Gregory served as consulting engineer to the Bureau, Mr. Hoover was engineer in charge of the Bureau, and Mr. Cornell was engineer in charge of the construction of the dam.

The O'Shaughnessy Dam, named for the late Jerry O'Shaughnessy, who was connected with the water works department of Columbus, Ohio, for nearly fifty years, and for many years was its superintendent, is a massive structure built across the Scioto River in Central Ohio, about 17 miles north of the city of Columbus, and was constructed so as to provide the city with a second large water-supply reservoir. The dam has a total length of 1,750 feet, including the approaches, and the masonry or spillway portion of the dam, over which water flows, is 879 feet in length. Crossing the dam, over the spillway section, is a reinforced concrete arch bridge of twelve spans. The bridge has a roadway of sufficient width for two lanes of motor vehicles, and on each side of the roadway is a sidewalk for pedestrians. The crest of the spillway section is 64 feet above low water in the river, and the total height of the dam from the rock foundation to the roadway level is 105 feet.

The Prettyboy Dam will be somewhat similar in construction to the O'Shaughnessy Dam. It will be built entirely of masonry, and will have a spillway section carrying a reinforced concrete arch bridge with roadway and sidewalks. It will be considerably higher than the O'Shaughnessy Dam but of less length. It is expected that the Prettyboy Dam will be completed in the fall of 1932.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

September 24, 1930. — The regular meeting of the Boston Society of Civil Engineers was held this evening in the Boston Chamber of Commerce Lounge, Boston Chamber of Commerce Building, and was called to order at 7.15 by the President, Lewis E. Moore. There were about 90 members present. Prior to the meeting about 75 members had supper in the Cafeteria in a space reserved for members of the Society.

The President announced that the minutes of the previous meeting (May 21, 1930) were printed in the September Journal and they were declared approved as printed.

The President announced the receipt of a bequest of $1,000 from the late Alexis H. French, the income of which is to be devoted to the library of the Society.

The President called attention to the publication of the report of the Committee on Flood Control in the September issue of the Journal, and stated that this was made possible by use of the income from the Freeman Fund through the courtesy of Mr. John R. Freeman.

The Secretary reported the names of those elected to membership by the Board of Government on September 24, as follows:

Grade of Member: Nathan Ginsburg,* George E. Meyer,* George N. Watson,* John W. Howard, Achille J. Presutti.

Grade of Junior: Jacob Ginsburg, Charles V. Maccario.

The President announced the death of the following members: James Mauran Betton, died May 29, 1930; James Rice, died May 29, 1930; Elliott Howes Gage, died July 13, 1930; Allen Hazen, died July 26, 1930; Ira Nelson Hollis, died Aug. 14, 1930; Daniel Webster Pratt, died Sept. 16, 1930.

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* Transferred from Grade of Junior.

It was *Voted* that the seventh paragraph of section 5 of the by-laws be amended to read as follows:

> Whenever a President, Vice President, Director, Secretary or Treasurer is to be elected, the Nominating Committee shall nominate one candidate for each office to be filled. Whenever a member of the Nominating Committee is to be elected, the Nominating Committee shall nominate two candidates for each office which is to be filled. Ballots shall provide space for the writing or pasting in of additional names by the voters.

This matter had been acted upon favorably at the meeting of the Society on May 21, 1930, and the affirmative vote at this meeting made action on this matter final.

The President presented Mr. George E. Seabury, chairman of the Engineering Societies of Boston, who spoke briefly on the relation of the Engineering Societies to the several constituent member organizations.

The President then announced that the author, Mr. Edward Smulski, of the paper on the "Design of Wind Bracing," was unable to be present, but that Mr. Edward H. Cameron had consented to present the paper, which was illustrated with lantern slides. Discussion of the subject matter was given by Messrs. Waldo F. Pike, W. W. Bigelow, E. A. Varney, Prof. F. N. Weaver, L. R. Hersum, L. E. Moore, S. C. Jemian and Prof. L. J. Johnson.

The meeting adjourned at 9.10 P.M.

EVERETT N. HUTCHINS, *Secretary.*

OCTOBER 15, 1930. — A regular meeting of the Boston Society of Civil Engineers was held this evening in the Lounge, Boston Chamber of Commerce Building, and was called to order at 7 P.M. by the President, Lewis E. Moore. There were about thirty members present.

The President stated that the minutes of the previous meeting (September 15, 1930) would appear in the October JOURNAL.

The President announced the death of the following members of the Society:

William Warren Cummings, died April 5, 1930, a member since February 16, 1898.
William H. Cronin, died August 14, 1930, a member since February 16, 1921.
E. Roland Simpson, died September 24, 1930, a member since October 16, 1901.

Committees will be appointed to prepare memoirs.

The Secretary reported the names of those elected to membership by the Board of Government on October 15, 1930, as follows:

Grade of Member: Thomas R. Camp, Donald F. Horton. *Grade of Junior:* Coburn M. Richardson.

Upon recommendation of the Board of Government as presented by the Secretary, it was *Voted* that the Board of Government be authorized to use the income from the Permanent Fund for the current year to such an extent as they deem necessary in payment of the current expenses of the Society.

This matter will be presented at the November meeting for final action.

The President then introduced Mr. Dudley Harmon, Executive Vice-President of the New England Council, who gave an interesting address on "The Industrial Situation in New England." The products of New England, and their marketing and the creation of a sense of unity in the endeavors to develop New England resources, were described.

The meeting adjourned at 9 P.M., after a rising vote of thanks to Mr. Harmon for his courtesy in addressing the Society.

EVERETT N. HUTCHINS, *Secretary.*

Sanitary Section

ANNUAL EXCURSION, SEPT. 26, 1930

SEPTEMBER 26, 1930. — Thirty-six members of the Sanitary Section met at Forest Hills at 1 P.M. Mr. Ralph W. Loud and Mr. Seth Peterson of the Metropolitan District Commission then conducted the party on an inspection tour of the new Neponset Valley sewer, now under construction by the Metropolitan District Commission.

Beginning at the lower end of the work, at its junction with the existing high-level sewer in Brook Road, Milton, the party viewed the several contracts, and saw the work in all stages of completion.

This sewer is of monolithic concrete, of varying sizes, with semi-circular invert and roof section, the depth being slightly greater than the width. At the lower end, the width and depth are slightly over 6 feet. Steel forms are used. Deep cuts predominate, and difficulty has been encountered from extremely hard soil in some sections and boiling sand in others.

The excursion left the upper end of the work at 4.45 P.M.

George G. Bogren, *Clerk.*

Designers Section

October 8, 1930. — A regular meeting of the Designers Section was called to order at 6.15 P.M. by the Chairman, Norman P. Randlett, in the Affiliation Rooms.

The reading of the minutes of the May meeting was omitted.

The first speaker of the evening was Mr. Maurice A. Reidy, Structural Engineer, who gave a talk on "High Buildings in Boston." Mr. Reidy used the new building at 75 Federal Street to illustrate many problems encountered by the structural engineer, including those imposed by special requirements of the tenant and by the architect.

The second speaker was Mr. D. W. Clapp of the firm of Blackall, Clapp & Whittemore, Architects, of Boston. Mr. Clapp described the future skyline of Boston as it is likely to develop under the Boston building law, comparing it favorably with that of New York and Detroit.

There were thirty-eight members and guests present.

The meeting adjourned at 8.30 P.M.

Lawrence G. Ropes, *Clerk.*

Highway Section

October 4, 1930. — The Highway Section attended the exhibit of the Activities of Various State Departments in the Commonwealth Armory at 2.30 P.M. on Saturday, October 4.

The afternoon was spent inspecting the exhibit of the Department of Public Works. Engineers of the Department of Public Works explained the design, construction, and maintenance of the highways and bridges, and also the laboratory equipment used for testing materials. The exhibit included an unusually fine model, fifteen feet in length, representing approximately 1,200 feet of road, showing types of pavement, bridges, etc.; a model of a traffic intersection controlled by traffic lights; a model of the projected Tyngsborough bridge; and a model of the separation of the highway grades on the Worcester Turnpike, such as at Cordaville.

Mr. P. H. Kitfield explained methods of making plans, estimates, and the system of bidding.

Mr. R. O. Spofford explained various types of bridges and the work of the bridge department.

Mr. F. T. McAvoy discussed construction, explaining the various types of highway pavements and the method of testing concrete beams.

Mr. J. H. Armstrong of the highway testing laboratory showed specimens and explained methods of testing materials.

Mr. H. W. Putnam showed motion pictures illustrating the surveying, construction, and maintenance of State highways.

Fifteen members and guests attended the exhibit. The inspection group adjourned at 4.30 P.M.

Ernest Mathers, *Clerk.*

Northeastern University Section

October 3, 1930. — The first meeting of the current year was held Friday evening, October 3, 1930, in Room 18H, with an attendance of seventeen.

The Section was very fortunate in securing the services of Mr. Van Ness Bates of the Warren Brothers Company as the speaker of the evening. This construction company is noted as "International Road Builders," and the speaker was thoroughly conversant with highway conditions throughout the world, specifically mentioning England, South America, United States, China, Russia and Poland. He showed how highways have developed through the ages from game trails down to our present roads. For future development in highway building, Mr. Bates proposes a system of express highways for any given geographical area.

The next part of the talk had to do with the financial end of road building. It is not the actual cost of a project that should concern the public, but the relative savings that will result. As an example of this Mr. Bates pointed to North Carolina's intensive road building program carried out during the last few years, which has resulted in an increased growth of 50 percent as compared with 6 to 10 percent in her neighboring States.

The third part of the talk was on planning; that is, the building of roads to take care of future needs and to act as part of a complete system of roads. The speaker dwelt briefly on construction features, and concluded his talk by showing some slides on the Cuban National Road, which Warren Brothers Company is building.

Refreshments were served and the meeting adjourned at 10 P.M.

R. H. VERNER, *Clerk, pro tem.*

APPLICATIONS FOR MEMBERSHIP

[Nov. 20, 1930]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

COPITHORNE, JOHN WILBUR, West Somerville, Mass. (Age 21, b. West Somerville, Mass.) He is a student at Northeastern University and during the co-operative periods is employed by Aspinwall & Lincoln. Refers to *W. J. Alcott, H. B. Alvord, C. O. Baird, E. E. Nightingale.*

ERICKSON, Jr., ARIOCH WENTWORTH, Swampscott, Mass. (Age 26, b. Swampscott, Mass.) Graduate of Massachusetts Institute of Technology in June, 1928; returned for graduate work during winter of 1928–29, became assistant in the department of civil engineering in winter of 1929–30, and received degree of S.M. in civil engineering in June, 1930. At present employed by Trimount Dredging Company. Refers to *J. B. Babcock, H. K. Barrows, H. T. Gerrish, C. M. Spofford.*

RANKIN, ELMER P., South Braintree, Mass. (Age 34, b. New Bedford, Mass.) Education: two years at Lowell Institute and now taking courses as a special student at Massachusetts Institute of Technology. Experience: 1914–15, draftsman with W. S. Johnson; 1916, with W. F. Gowing as draftsman; 1917, with Stone & Webster Engineering Corporation as draftsman; as designer, with Fay, Spofford & Thorndike, 1918–19; and with Lockwood, Greene in same capacity, 1920–21; designer, with Stone & Webster, 1924–25; and structural engineer with Coolidge, Shepley, Bulfinch & Abbott, 1925 to date. Refers to *W. W. Bigelow, F. M. Meyer, B. A. Rich, C. E. Tirrell, E. A. Varney.*

MINICHIELLO, A. ALBERT, East Boston, Mass. (Age 22, b. Boston, Mass.) Graduate of East Boston High School, and entered Northeastern University in September, 1927. During co-operative periods has been employed as a rodman by A. C. Peters and New England Power Company; as a transitman by B. F. Snow of Boston and N. B. Snow of Barnstable, Mass. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, C. A. White.*

DEATH

PRATT, DANIEL W. . Sept. 16, 1930

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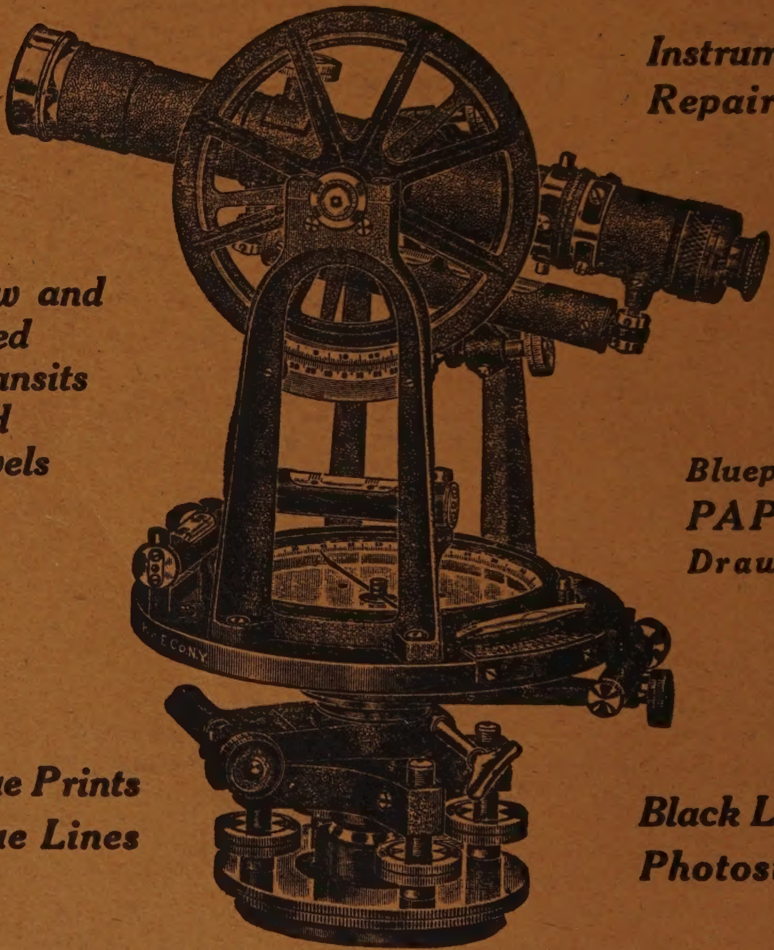

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